AMERICAN SOCIETY OF CIVIL ENGINEERS.

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CLXIX.

(Vol. VII.—September, 1878.)

THE DANGERS THREATENING THE NAVIGATION OF THE MISSISSIPPI RIVER

AND THE RECLAMATION OF ITS ALLUVIAL LANDS.

By B. M. HARROD, C. E., Member of the Society.

Read at the Tenth Annual Convention, June 18th, 1878.

When the operations of civilization are first extended into new fields, the pioneer, while acquiring rights and benefits, inevitably assumes important responsibilities. If he locates his farm upon the fertile banks of a stream, on which he proposes to transport his wealth to market, he soon disturbs the subtle relations existing between its supply and discharge. If he follows the ravine up the mountain side and sinks a shaft, the accumulation of debris therefrom, unless removed, causes freshets

that will sweep him and his improvements away in a night. If, on a deep and safe harbor, he concentrates radiating lines of transportation, and establishes costly facilities for protecting and handling an accumulation of merchandise, he, at the same time, develops forces which will steadily impair the advantages which first attracted him. Nature will form no co-partnership with man. If he enters her virgin domain, to reap the advantages of civilization and science, he assumes the duty of replacing the natural status by his scientific care and forethought.

So the improvement of the navigation of the Mississippi river and the reclamation of the alluvial lands of its valley are not peculiar as engineering problems, because their theoretical treatment would be comparatively clear and simple were it not for abnormal complications arising from the disturbance of established natural relations. Could basins be built on its tributaries, to hold in reserve its supplies, to prevent their simultaneous discharge, to moderate the violence of excessive floods and at regulated periods to send down just sufficient water to keep the river in good boating order, and not enough to cause a velocity that would demolish the banks and levees, there is little doubt but that, in a few years, the river would permanently locate its bed in a succession of tangents and curves, determined by the equilibrium between its centrifugal momentum and the stability of the material of its banks.

That this condition was approximated, to some extent, before the settlement of the great valley is probable from historic facts relative to the increased magnitude of cavings and frequency of cut-offs, which will be given hereafter.

It is certain, however, that at this day the difficulties of the restoration of a better condition of the river are increasing. The enlarging area of cultivation has, by furrowing and breaking up the crust of the soil, made drainage and absorption more rapid and complete, by which greater floods are precipitated with more violence—has, by clearing the banks, deprived them of the protection of the thickly interlaced roots and the overhanging and trailing branches of the dense border growth—and has, by the extension of the levee system, had a tendency to raise the surface and increase the slope until a velocity is given before which the treacherous material of the banks wears and caves. Thus, while impetuosity has been given to the attacks of the floods, the banks have been deprived of the natural defences which their fertility reared for them.

The caving is immediately produced in two ways—which are characteristic of sand or clay bank. When the changes in the course of the river throw the force of the current against a bank of recent formation by deposit, it wears to a steeper slope than the limited tenacity of the material will maintain. This is the simpler and more rapid process.

When, however, the bank is of older alluvial formation it is found stratified with tough blue clay and flat horizontal pockets of quick-sand, shells or other treacherous material. Whenever one of these pockets is worn into, the contents exude from pressure, or are washed out by abrasion, and the leaf of clay above, deprived of support, breaks off and tumbles its load into the river. The profile of such banks shows the blue clay strata with an easy slope, and those of quick-sand or shells very steep, or even with an overhang.

An explanatory diagram accompanies this description.

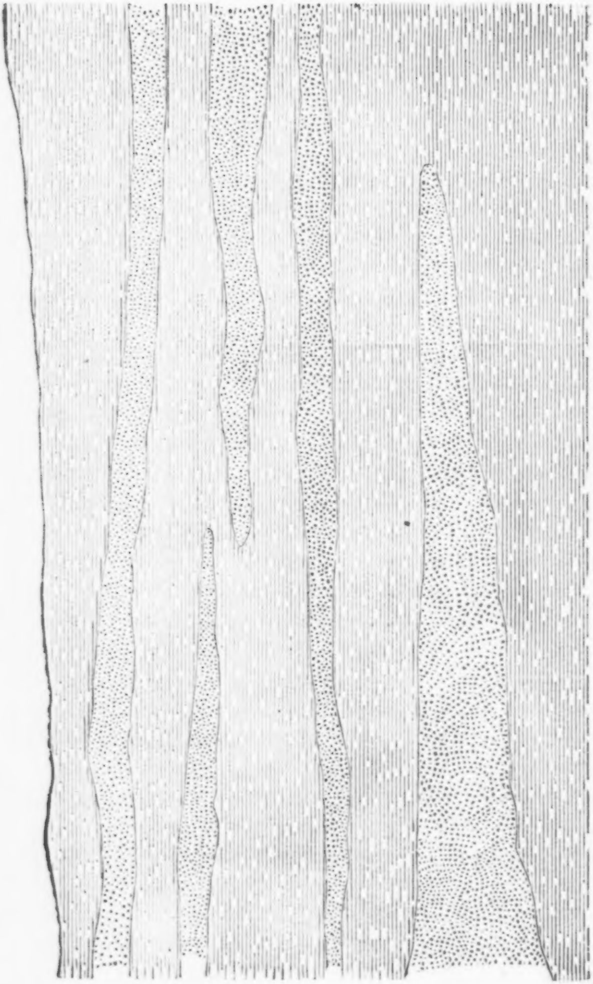
Thus, all the processes necessarily accompanying the settlement and cultivation of a new country have served to render more difficult of attainment the regimen which prevailed under the normal operation of the functions of nature, viz. : rainfall, surface drainage, absorption, slope, velocity, and discharge, and have given to the current of the Mississippi River an unnatural and destructive activity.

The curving banks which formerly served to turn the momentum with which the waters swept down have now been distorted by more rapid and excessive floods, which, deprived by levees of temporary relief from the swamp basins, have raised the surface, and increased the velocity to an extent that the banks are unable to resist.

At many points this process in the bends above and below has reduced the isthmus between, until it was no longer able to hold back the impending cut-off.

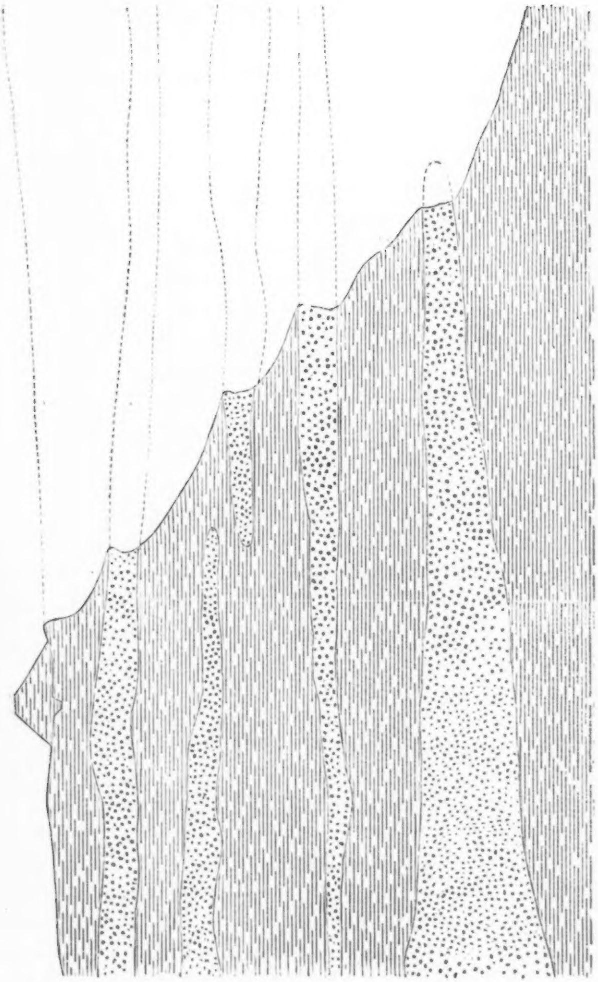
Thus a current already dangerously rapid from abnormally increased volume and slope is intensified by a reduction of length.

To this unnatural activity of the forces of the river, stimulated by a disturbance of regimen, and culminating in cut-offs, is to be attributed the deterioration of navigation and the destruction of levees. Notwithstanding the proof that causes have been at work to produce more rapid and excessive floods, and that the increased caving of banks, frequency of cut-offs, and obstructions to navigation are directly traceable to these causes, nevertheless, plans of improvement are seriously discussed which involve measures productive of a further increase of flood velocity and



SECTION THROUGH ALLUVIUM.

Lined Work indicates Clay. Dotted Work indicates Sand.



SECTION SHOWING ENCROACHMENT OF CAVING BANK OF RIVER ON ALLUVIUM.

volume, a prolonged contention with the river in its ceaseless effort to reduce its slope and recover its normal length, and delay and difficulty in the attainment of a regimen.

The existing functional derangement will be more readily appreciated and firmly established by the following facts in the history of the great river.

Many centuries are required to obliterate the traces of a cut-off. The earliest to which any date is attached is the Fausse Riviere cut-off of Pointe Coupee, which occurred in 1722.

The banks of this artificial lake are still well defined, and its waters, at the flood period, still connect with the Mississippi.

In the lakes and old rivers on either side of the Mississippi, from the Tennessee line about six hundred miles down to Pointe Coupee, we find the evidence of about twenty-eight cut-offs. Undoubtedly this record is complete for many centuries.

Yet of the twenty-eight cut-offs, nine, or about one-third, have occurred within the past forty-five years. This epoch of forty-five years has also been marked by two other important changes in the characteristics of the river: the clearing and leveeing of the banks for the greater part of this six hundred miles, and the extension of the bars and obstructed navigation from Plum Point, one hundred miles below Memphis, to Lake Providence, a distance of nearly two hundred and fifty miles, included in this six hundred miles where levees have been built and cut-offs permitted.

If these co-incident events do not establish clearly enough the connection between a disturbed regimen and the disasters to reclamation and navigation, let us examine a cut-off whose history is so recent, and whose record is so ruinous as to have attracted and fixed the attention of all observers of the hydraulic phenomena of the Mississippi River.

During the flood of 1867 the isthmus connecting what is now known as Davis Island with the Mississippi shore about twenty-seven miles below Vicksburg was broken through. The distance around this point was about twenty miles, throughout which the fall was about 3 inches per mile, or a total of five feet, which was condensed by the cut-off in the one quarter of a mile through the cut-off. Half of this increased slope was distributed down the river to below Glasscock's Bend, a distance of one hundred and fifteen miles, and caused a disturbed regimen and an accelerated velocity to which is attributable the follow-

ing record of ruin between the date of its occurrence and the present time. It is confined exclusively to the Louisiana shore. It is fair to assume that the Mississippi shore suffered equally :

Location.	Length in Feet.	Maximum Width in Feet.	Cubic Yards.	Acres.	Cubic Yards Levee.
Point Pleasant	11,440	6,160	100,000,000	1,075	383,801
Wilson's	8,800	2,640	30,000,000	330	320,962
Hard Times	14,000	1,000	15,000,000	175	496,791
Boudurants	30,000	10,000	25,000,000	300	242,615
St. Joseph	15,000	3,000	40,000,000	500	44,986
Kempe	16,000	5,000	100,000,000	1,250	1,315,495
Waterproof	18,000	2,600	25,000,000	300	497,296
Marengo	25,000	5,000	120,000,000	1,500	366,075
St. Catherine's Bend	10,000	2,400	20,000,000	275	114,339
Glascock's Bend	38,000	2,000	75,000,000	850	732,960
Total	186,240		550,000,000	6,555	4,515,320

In this table is not included the levee work done by parties other than the State Engineers, which is of a considerable amount.

No details are given of the damage above by this cut-off, although it was equally evident and extensive, because it was confused with that caused by the Terapin Neck Cut-off, which occurred in the previous year about sixty miles above Davis Island.

These reflections on the causes and magnitude of the difficulties to be encountered in securing the unbounded wealth of the Valley of the Mississippi, disclose the outlines of a plan by which they can be encountered and defeated.

This, or any other plan, must, however, be adopted under the explici and intelligent recognition of two inevitable complications :

1st. The continued clearing for cultivation of the banks of the Mississippi and its tributaries will continue on the most extensive scale ; and

2d. The levee system will be pushed to completion. The first o these obstructive elements is practically unlimited in its duration. The second is not so. It is purely a financial question whether the system shall be completed in two or twenty years.

But inasmuch as the extended construction of levees will have a tendency to disturb the regimen whenever it may be undertaken, they should be eliminated from the problem at the earliest possible moment, and a complete levee system should be assumed as the first step in all projects

of improvement. A judicious location of these works will allow for sufficient prolongation in the bends to give the required reduction of slope, and will be governed by the experience of the engineer, and formulæ, in which the volume and velocity, increased by the completion of the projected work, and the angle of incidence, are opposed to the stability of bank, as composed of clay or sand.

The necessary height has never yet been determined by experience, for great floods have always found a point, vulnerable from carelessness or fraud, before they culminated.

The height recommendations of the U. S. Commission of 1874 are more than safe, for while all the elements of danger are recognized, no credit is allowed for hydraulic principles as well established as those on which are based their precautionary recommendations. Thus, whenever the retention of all the water within the levees would require that they should be built eleven feet higher at Lake Providence than the flood of 1858, then the depth would be increased eleven and one-half per cent., the volume in a still greater proportion, and the slope upwards of eight per cent. from Lake Providence where the flood wave culminates to zero at tide water. These new factors would generate a velocity and discharge that would forbid the verification of the estimated increase of flood level.

With the completion of the levee system, the question of outlets loses most of its importance. The objections to new ones consist in the greater slope required for a smaller volume, and in the continuance of the present disturbance of regimen. Of those now existing the Atchafalaya is now mainly supplied from Red river, the Lafourche is of insignificant dimensions, and the Jump and Cubits crevasse are too near the mouth of the river to affect the levees or navigation.

But certainly the most important measure of all for securing the great objects proposed is, the absolute prevention of "cut-offs" at any cost, by the thorough *revetment* of the approximating concave banks.

Subsequent measures must be conservative, tending to remove all disturbing elements. Among them may be mentioned the encouragement of a dense growth on the banks between the levee and the river.

The penalty of any forcible treatment is immediate reaction on the weakest part of the bed. Before it will be experimentally ascertained whether the bottom is of clay, "almost like marble," or whether "it will not resist the incessant action of the current," the banks will be

melting before the increased current unless protected by miles of revetment which the experience at Cairo, and the estimates at Memphis, Vicksburgh, and New Orleans show will cost $100,000 per mile.

In the report of Col. Simpson, U. S. Engineers in 1875, on the improvement of the navigation of the Mississippi river, the outlines of a system are given with clearness and ability. It is established that the low water channel referred to in speaking of the improvement of navigation and the line of greatest flood depth do not coincide, nor is it the result of the operation of the forces of the previous flood. On the contrary the subsidence of the flood leaves the channel in an impaired condition, obstructed by a succession of sand waves or bars. The crests of these bars mark the thread of the flood current by their composition of the heaviest material. The feeble forces of low water avoid this line in excavating their channel. It is therefore, important that the scanty resources of the river at this season should be carefully husbanded, and that works designed to aid in establishing a low water channel should not increase the flow over a bar, lest obstructions higher up the stream be developed. The effort should be to give a more efficient working section to the channel while preserving its capacity of discharge.

It would appear that the conclusions to be drawn from the foregoing remarks are, that the first work to be done on the river is the prevention of cuts-off and the completion of levees. Then, under this discipline, the river should be allowed to acquire the relations of slope, length and velocity imposed upon it by hydraulic law.

This condition will be comparatively permanent, and, under it works for the improvement of its navigation can be pushed with impunity under a system that combines with the scouring power of an accelerated current, proper precautions to protect the treacherous banks, and direct its action against the bottom. Let it never be forgotten that "a great river is impatient of restraint; it can be led, but it cannot be driven."

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CLXX.

(Vol. VII.—September, 1878.)

BRICK ARCHES FOR LARGE SEWERS.

By R. HERING, C. E., Member of the Society.

Read at the Tenth Annual Convention, June 18th, 1878.

The City of Philadelphia, having many miles of large brick sewers with diameters as great as 20 feet, offers a favorable opportunity for studying, among other interesting questions, the stability of arches with long axes. And it has been especially facilitated by the occurrence of some failures, such as flattening at the crown, and spreading at the spring, or entire collapse. The usual design was a circular ring of brick, held in position by abutments, and spandrel filling of rubble masonry, the dimensions being calculated according to the ordinary empirical formulæ. An inquiry into the conditions of stability by means of the method of Graphical Statics showed that even with a fair execution some of the arches could not permanently resist the pressure. Instead of increasing the dimensions, a careful study led to the adoption of a different plan, especially for cases where the horizontal thrust cannot at first be safely resisted by the earth on the sides, but must be taken up by the abutments, and transferred to the foundation. During

the last two years several miles of arches from 5 to 20 feet span have been built according to this plan and under these conditions, with perfect success.

As the construction is somewhat different from the usual practice, I believe it a proper subject for discussion.

Before stating the main points, I will describe two breaks in our largest sewer, built about seven years ago, and point out the reliability of the principles which form the foundation for the present design, by comparing theoretical conclusions, as far as possible, with the practical results.

Fig. 1 (Plate XXXII) represents a half-section of the Mill Creek sewer 20 feet in diameter at the spring. The dotted lines in the arch represent the original section and the full lines a section as standing adjacent to a break. The semicircular arch is built of two concentric rings of brick, and the abutments are built of rough rubble masonry, shaped as shown in the figure. The workmanship was not of the best; the mortar is poor and scanty. Testing the original design by drawing in the line of pressure (indicated by a dash-dotted line) from the diagram of forces in Fig, 1*a*, it will be noticed, after equalizing the danger of breaking at all the joints of rupture, that this line is not confined to the kernel of the section, that is, the middle third of the joint area.

What will be the result?

When the centre of pressure falls within the kernel, the neutral axis lies outside of the section, and causes compression on the whole area. As it passes out of the kernel, the neutral axis enters the section, when tensile strains, acting on one side, will tend to crack the mortar and open the joints; and the compressive strains, acting on the other, and being resisted by a smaller area, will cause a proportional increase of the unit strain.

Now, in Fig. 1 the line of pressure intersects the joints of rupture at the crown and haunches of the original design at $\frac{1}{10}$ of the thickness of the section, locating the neutral axis at $\frac{3}{10}$. Therefore, $\frac{7}{10}$ of the section are under tensile strain, while all the compression is confined to the remaining $\frac{3}{10}$.

The existence of the *tensile* strains can be seen on the work in the actual opening of a part of the joints of rupture at the crown and haunches, as indicated in the figure. As the strain in the most com-

pressed fibre is twice as great as the strain per unit of joint area,* we can also readily examine into the condition of *compression* at those joints. From Fig. 1*a* we measure the thrust at the joint of rupture in the haunch as 13 900 pounds per running foot. The pressure being distributed over $\frac{3}{10}$ of the section, or $5\frac{4}{10}$ inches, will give a strain of $\frac{2 \times 13\,900}{12 \times 5\frac{4}{10}} = 429$ pounds. According to Trautwine, cracking and splitting of brickwork occurs at about 400 pounds, which shows an unsafe condition in this case. And, as this strain proportionately increases the nearer the line of pressure approaches the edge, we should justly expect to find extensive chipping of the bricks near the theoretical joints of rupture in the section drawn out with full lines. An examination of the sewer, where this section is approached at many points, shows it to be actually the case.

The result of the original position of the line of pressure then, will be a constant tendency of the arch to rotate at the joints of rupture in a direction from the compressive to the tensile strains, thereby causing a sinking at the crown and a pushing out at the haunches. This change of form has also taken place in a greater or less degree throughout the whole sewer.

Lastly, the bed joints of the rubble and brick masonry at the springing line deviate from twenty-five degrees to twenty-nine degrees from a perpendicular to the line of pressure, showing an unsafe resistance against sliding, especially as the masonry was poorly built. The horizontal thrust has therefore very generally caused a spreading of the arch, though it is not indicated in the figure.

Fig. 2 represents a half section of the Cohocksink Creek sewer, with a horizontal diameter of 18 feet 6 inches. The arch is elliptical and built of brick, with a thickness varying from 18 inches at the crown to 3 feet at the spring, and the abutments are built of rubble masonry, as shown. The original section, marked with dotted lines, has changed its shape more or less, and adjoining a break it appeared as shown by the full lines. The workmanship, materials and mortar were good. It was built about eight years ago.

Testing this design by drawing in the line of pressure from the diagram of forces in Fig. 2*a*, the same discordance with the laws of stability will be found as in Fig. 1.

* See Woodbury's Theory of Arches and Culmann's Graphical Statics.

The line intersects the joints of rupture at $\frac{1}{6}$ of the thickness, instead of at $\frac{1}{3}$, locating the neutral axis in the middle of the section ; therefore, the constant tendency to rotate has here also caused the crown to settle and the haunches to push out. As the weight over the arch, however, is much less than in Fig. 1, the thrusts are smaller, and the centre of pressure can approach the edge closer before the bricks are crushed, which in this case would occur at a distance of about $\frac{3}{4}$ of an inch from it.

Cracking and splitting of the bricks can be noticed all along the theoretical loci of the joints of rupture, and likewise the corresponding openings on the other side of the neutral axis, as shown in the figure, and as theory leads us to expect.

Finally, the line of pressure intersects the perpendicular to the bed-joints in the springing at a larger angle even than in Fig. 1. As the resultant at that point is 9 800 lbs., and assuming the co-efficient of friction for masonry to be .6, we find a horizontal resistance of only 5 880 lbs. against the horizontal thrust of 5 590 lbs. And this small excess of 290 lbs. is more than balanced by the pressure of water, when the sewer is running nearly full, which always occurs during a heavy storm at high water. Consequently, the entire sewer has spread from 6 inches to 12 inches, until the earth on the sides has become sufficiently compact to prevent further movement and to resist the line of pressure at a higher point.

Such failures as shown by these two sewers, and the fact that the conclusions arrived at from the application of the method of Graphical Statics were so truly confirmed by the actual results of our experience, led us to adopt a different design, which it was intended should be based entirely on principles of this method. In arranging it, the following main points were carefully considered :

I.—At no point should the material be subjected to more than a safe fraction of its ultimate crushing strain. Assuming the safe resistance of brickwork at 80 pounds per square inch, a very convenient formula has been :

$$\text{Least thickness of arch in inches} = \frac{\text{Pressure at joint of rupture in lbs. per running foot.}}{480}$$

which is deduced from the fact that the strain in the most compressed fibre is twice as great as the average strain per unit of joint area.

II.—The bed joints of the bricks and stones should everywhere be as nearly as practicable at right angles to the direction of the line of support, and not deviate from it more than a safe fraction of the resistance against sliding will admit.

III.—The line of pressure must be situated in the kernel or middle third of the arch, to prevent any tendency of rotating. It must also intersect the foundation at a point sufficiently far from the edge to be safely resisted, on account of the rough nature of the rubble work.

When the earth on the sides is solid enough to resist the resultant of pressure, it can of course take the place of the abutment, and the pressure line does not require to be enclosed in mason work down to the foundation, but can pass out into the earth at any point.

IV.—The different materials should be disposed of in such a manner that each may serve for the purpose to which it is *best* adapted.

The necessity of carefully curving the line of pressure in the arch demands well defined joints, thorough bearing surfaces, and homogeneous masonry. Therefore, well-built brickwork, being the best for this purpose in our case, was planned to resist alone the entire pressure, as far as it is curved. When it becomes practically straight in the abutments, and distributed over a larger surface, rubble masonry will then answer perfectly. As the spandrel filling has no other duty to perform, according to theory, than to furnish a uniform vertical pressure upon the arch, earth alone, when well compacted, can entirely fulfill it. The gain of a greater weight by using rubble masonry and thereby slightly reducing the width of foundation, may sometimes be economical, but it is usually more than balanced by its own greater cost. If the rubble masonry is to take part in resisting the line of pressure, it can only do so when forming an auxiliary arch of itself, outside of the brick arch ; otherwise it acts only as weight. Wherever the filling is light, and the live load great, stone spandrels would *stiffen* the arch against a concentrated pressure. But this extra stiffening is not necessary where the live load can be neglected in the calculation, as is usually the case.

V.—The brickwork is to be bonded, so that the line of pressure will pass through the middle of headers as much as possible, to prevent separation of the rings.

The design resulting from the consideration of these five points is shown in Figs. 3, 4, and 5, which represent three sewers, built one and two years ago. A careful examination during construction, and since, shows

perfect stability. No cracking at the crown, chipping of bricks at the haunches, nor change of original shape have taken place.

Besides seeming to be a more rational distribution of the material, and therefore more reliable in its stability, this plan is also more economical, as it requires less material to give the same strength as the old one. To secure these advantages, however, the abutments must be built carefully and solidly, so that there is no possible uneven settling, which might cause rotation around the springing line. The ring joints in the brickwork, as well as the others, must be well filled with mortar to prevent separation ; and the earth filling in the spandrels must be uniformly and well compacted in order to answer the supposition of theory that the entire weight acts like its supposed resolution into separate weights as indicated by the division into vertical laminæ.

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DISCUSSION ON PAPER CLXX.

BRICK ARCHES FOR LARGE SEWERS.

By E. S. CHESBROUGH, W. MILNOR ROBERTS, R. HERING and F. COLLINGWOOD.

E. S. CHESBROUGH:—Mr. Hering has sent me two sketches with a brief statement of the main points of difference between the "old" and the "new plans," from which I learn that in some respects the mode of constructing sewers in Philadelphia has been peculiar. I am ignorant of the circumstances which led to the introduction of the "old plan" at first, but it seems strange to me that it should ever have been adopted for general purposes, whatever might have been advisable in some special cases. The "new plan" is substantially that which has been known to the profession, in connection with the stability of arches, for a long time. The arrangement of the joints between the arch stones, placing them at right angles to the line of pressure if that line can be readily determined, must be of decided advantage. If the sketches sent me are drawn to a scale, the thickness of the arch was too light, by the "old plan." The arches of the large London and Paris sewers were made

much thicker than those of the Philadelphia sewers. On the other hand, many, if not nearly all of the older London and Paris sewers have far less backing, or masonry outside of the arches, and it seems strange that so much should be needed in Philadelphia, unless the peculiarity of the soil requires it.

When sewers are laid deep enough to be beyond the disturbing influences of other works, and are built in simple arches of sufficient thickness and good materials, they have been known to stand without change of form, for generations, with no backing of masonry. The necessity for this, if it exists, must be determined by the circumstances of each case. It is not only an extra cost in itself, but requires additional cost for excavation, back-filling and repaving.

W. Milnor Roberts :—More than a year ago I examined Mr. Hering's plan, and his arrangement of the brick arch and haunches in connection with sewers in the City of Philadelphia, having diameters of 13 feet, 15 feet and 16½ feet, respectively, and I was satisfied that it was safer than the plan of the Cohocksink Sewer, which I had occasion to examine at a point where it had failed, and where the arch had fallen. I also had occasion to examine the circular sewer, nearly a mile in length, in which occur the diameters of 13, 15 and 16½ feet.

I understand Mr. Hering to limit the whole question to the case where the earth on the sides is originally not firm enough to resist the resultant of pressure ; where masonry abutments of some kind would be required.

The brick inverts of these circular sewers were of the thickness of one brick, lengthwise, and these were laid upon a cradle of rubble masonry extending laterally to the outside of the rubble abutments, and outside of the line of pressure of the arch. I did not consider that any rubble masonry was needed below the lowest part of the brick inverts, where the sewer rested upon a solid timber and plank foundation. I deemed it an injury, rather than an advantage ; but that does not necessarily affect the question of the stability of the arch and its abutments. If I had planned a double sewer, where the twin 13 feet sewer is built, *upon a solid platform*, I would not have had circular sewers at all ; but semi-circular arches of a little less span, with side walls.

Where semi-circular arches rest upon abutments, without inverts, it is true, there is a greater pressure transmitted from the arch through the abutments than where inverts are used ; to the extent, of course, of

whatever portion is transmitted through and sustained by the inverts; but there is no saving in the quantity of masonry; because the brick invert, *in the case in question*, has to be supported between the middle of the sewer and the abutments proper by masonry.

Not being called upon to do so, at the time I examined the fallen Cohocksink sewer, I made no special investigation as to the immediate or proximate cause or causes of its failure. Mr. Hering's exposition of what he believes was the cause appears to me to be sustained by his theory.

I was satisfied at the time, and I am still satisfied that Mr. Hering's arrangement of the brick work was much stronger than sewers of the dimensions mentioned are usually built, but at the same time I expressed the opinion to Mr. Hering that the top course of brick which he had added was unnecessary; for the reason, that the arches, according to theory, and according to my experience in practice were thick enough without it.

That which I approved of was the principle of somewhat increasing the thickness of the regular brick work on the haunches so as to bring the line of pressure and maintain it near to the middle of the brick masonry, and so as to diminish somewhat the quantity of the rubble masonry abutments near the top. I also approved of the arrangement by which the bed-joints were laid nearly perpendicular to the pressure line in the middle third of the brick work. Bonding the bricks so as to throw the line of pressure through header courses to prevent separation of rings is also an advantageous arrangement, adopted by Mr. Hering.

An equilibrated arch may not be a circle on the intrados, if the structure be required to sustain a heavy dead load, level on top, which is sometimes the case with sewers; but the sewer being circular, with an invert, the arrangement of Mr. Hering's seems to be an approach to equilibration; having little tendency to break in one part more than in another, and is on that account commendable.

R. Hering.—In reply to Col. Roberts, I would like to state that the thickness of the arch in Fig. 1, was determined by some old empirical rule, and as my examination has shown, and as Mr. Chesbrough writes, it is too light. The increased thickness in the new designs, which Col. Roberts thinks was unnecessary in one case, is in exact accordance with the latest empirical formulæ of Trautwine, Rankine and others.

The safe working load for brickwork is given all the way from 80 to 280 pounds per square inch. We have been assuming it at 80 pounds, and find this satisfactory

F. Collingwood.—Our experience on some small brick arches in the approaches of the East River Bridge confirms what Mr. Hering has said in reference to the use of headers rather than of ring courses in an arch. I am inclined to think that a judicious study of the band to be employed is *quite* as important as the discussion of the general form.

No doubt many of you read some time ago of the failure of a small brick arch in the Brooklyn approach whereby one man lost his life. After the accident I was called upon to make a critical examination and report upon the same.

The general arrangement of that work is a series of heavy piers about 100 feet long, and faced with stone, and running across the bridge. These piers are to be connected by heavy arches over which the roadway is to pass. Each pier is to be a double arcade of ten arches, five on each floor. A portion of these are shown in the drawing on page 263, and it was the arch (*a b*) which failed.

The arch had a span of 13 feet; rise, 16 inches, and thickness of 20 inches; and was laid up in five distinct rings of brickwork, or built "rowlock," as it is called. The length of the axis was 6 feet. It abutted on limestone skew-backs, and at the time of the failure about 8 inches of brickwork above the crown had been laid on nearly uniformly. The abutment for 11 feet below the spring was 7 feet wide and 6 feet thick down nearly to the line (*f d*).

At the time of the accident the masons were at work on top of the arch, when the foreman saw a crack open across the top of pier at (*b*). He instantly warned the men and they ran to the ladders. By the time they had descended to the floor below, the end (*b*) had settled about 6 inches, and was turning on (*a*) as a pivot. By the time they reached the ground the arch was falling. It struck the lower arch and broke it in two, and all fell together, leaving the abutment (*g a*) standing, but showing cracks, approximately at (*c d*) and (*g h*), (I have lost the sketch I made at the time). A movement of an inch was probably enough with the crushing of the mortar which would occur, to let the arch slip through.

After making a careful computation of weights I applied Dr. Scheffler's graphical method, taking first the case where the line of pressures was everywhere within the middle third. This line pierced the line *d f* at

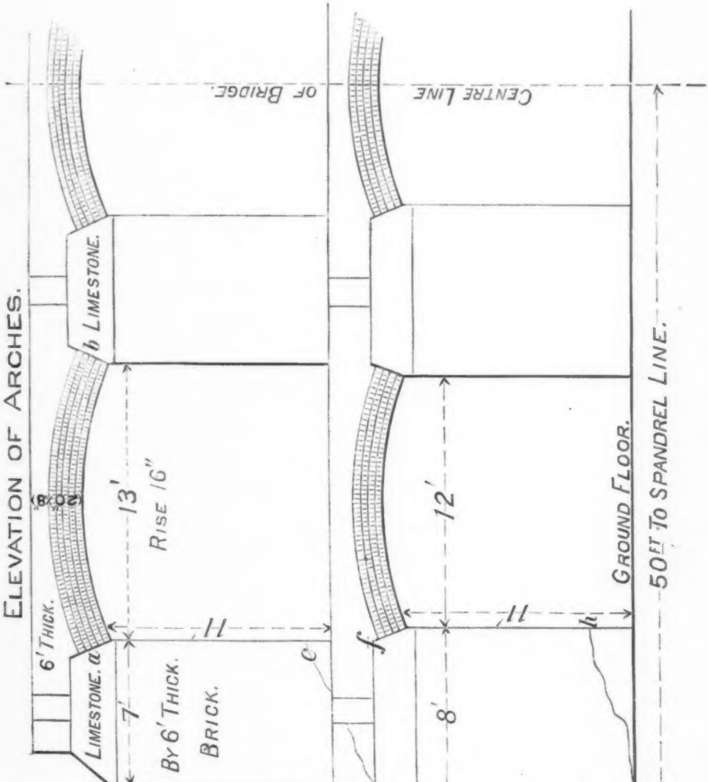
about 13 inches only from (*d*); and by trying the moments of the thrust and the pier at *d* as a centre, they were about as one to one and one-tenth. Trying the possible case of the line of pressures approaching to within two inches of crown and two inches of intrados at spring (as the brickwork would still be strong enough to resist crushing, the pressure being but 50 to 60 pounds per square inch if resisted by one ring only), the line of pressures fell within the base at the *ground floor;* and the moments about *d* were about as 1 to $1\frac{5}{10}$. As this brickwork had been set in strong cement mortar it had considerable strength as a beam; and I considered the investigation as proving thus far, that failure could not have occurred by the arch acting as a whole.

There was no evidence of compression in the outer joints of the abutment. There was, however, some evidence of the bricks just below (*a e*) having slipped a little on their beds while the mortar was but partly set. This led to the supposition that there might have been just enough movement to leave but a partial bond or adhesion of the mortar between the several rings, and that they had acted more or less as a series of arches. In *this* case the line of pressure could *not* have passed from ring to ring, but the resultant of all would be approximately through the central ring. Trying *this*, I found that the thrust would be nearly double, and the resultant pressure would run out a little above the line (*d f*). The slowness of the movement shows that the abutment was barely overbalanced. Other piers precisely similar had stood. The centre had been removed probably a little too soon, although it had stood two or three days and did not fail until the men were working on it; but I am satisfied in my own mind that the arch would *probably* have stood had it been bonded through sufficiently to transmit the thrust from one ring to the next.

Mahan says: "The entire arch should be divided into several portions, by joints running entirely through from the soffit to the back, the brick being laid in these successive portions in shells (or rings); and in blocks, with joints running *entirely through the arch from the* soffit to the back. Any bond may be adopted for the portion laid in shells. The blocks in which the joints run entirely through should not consist of more than three or four bricks in thickness, estimated along the curve of the soffit."

In other words, he would form masses which would approximate in form to the voussoirs used in stone work, with thinner strongly bonded masses between them.

I ought to say before closing that the exterior arch abutting against this is abundantly strong to take all thrust, and that *this* arch would probably be safe itself when it had the weight of the main arch and roadway upon it.



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CLXXI.

(Vol. VII.—September, 1878.)

FALL OF THE WESTERN ARCHED APPROACH TO SOUTH STREET BRIDGE, PHILADELPHIA, PA.

By D. McN. STAUFFER, C. E., Member of the Society.

Read September 4th, 1878.

It is a well-accepted engineer's maxim that failure is more instructive than success, and if the following account of the disaster at South Street Bridge proves interesting to engineers generally, and serves as a warning to the younger members of the profession, the purpose of the writer is fulfilled.

The lesson specially taught by this experience adds but one other to the many preceding it, and treats of the vital importance of an exhaustive examination, *at any cost,* into the material underlying foundations.

In this particular case "economy" in the preliminary explorations caused the complete failure of otherwise exceptionally good work.

The description is as much in detail as possible for the writer to make it, for though he was the engineer in charge of the original construction, his connection with the bridge was entirely severed on its completion in February, 1876. Since that time the work has been in other hands, and he can only describe what happened after his departure from such data as he could gather from those now connected with it.

The portion of the bridge destroyed was made up of nine segmental stone arches—43 feet 6 inches span and 14 feet rise, with a width of 55 feet. The arch piers—eight in number—were 55 feet long, 5 feet 6 inches thick and 12 feet 6 inches high. These piers were built of granite ashlar, in courses 17 inches to 26 inches rise. The masonry foundation under the piers was 9 feet high, with a base of 10 feet by 60 feet, the material limestone. The arches were laid in hard brick; they were 24 inches thick, with ringstone and skewbacks of granite. The haunching was limestone rubble masonry, and was carried up level with line of intrados of arch at the crown. The spandrel walls were limestone, faced with Trenton sandstone ashlar, and were surmounted by two 12-inch granite coping courses. All masonry was laid in hydraulic cement mortar, the brick arches in one cement to one sand, all other portions in one cement to two parts sand. To obviate settlement, a considerable portion of the filling in between the spandrel walls was small stone and quarry refuse; the remainder was gravel. North River flagging was used for the footwalks and granite for the curb. The roadway was laid with granite blocks.

These nine arches and eight piers fell, Sunday morning, February 10, 1878, the original cause being the sinking of some of the piles under one arch pier.

The ground underlying this approach is an alluvial deposit—treacherous and unstable in character; it is flooded to a depth of about two feet at high tide.

At the west bank of the Schuylkill this deposit is about 60 feet deep to the rock beneath, (a micaceous gneiss). The rock rises rapidly toward the west, and comes very nearly to the surface at a point just west of the arched approach. Overlying the rocks is a stratum of very hard gravel, about 20 feet thick at the river, and above this gravel is mud, which becomes more dense as the depth increases, passing into a tough clay before it reaches the gravel. Running through this mud deposit at different depths are strata of very hard gravel from six to eighteen inches thick.

Soundings were taken over this ground with a pointed iron rod, screwed down by the combined effort of four men, applied to an iron adjustable cross-bar bolted to the rod. With this apparatus it was impossible in all cases to reach the rock, owing to the extreme hardness

of the gravel strata. At the river the depth to rock was arrived at by data obtained in sinking the pneumatic cylinders forming the river piers.

"The want of an appropriation for any such purpose," unfortunately prevented a more thorough and elaborate investigation by means of "test wells."

When the writer was appointed engineer in charge, in 1870, he found a contract already entered into and work commenced on the plan of approach described. The specifications of the contract called for "pile foundations or timber platforms," when rock was not obtainable.

Numbering the arch piers from the west toward the river, the west abutment and Pier No. 1 were on timber platforms on gravel, the gravel stratum rising very rapidly between Piers Nos. 2 and 1. Pier No. 2, the settlement of which caused all the trouble, and Piers 3, 4, 5, 6, 7, 8, and the river abutment, were founded upon piles.

After piles of pine and oak had been shattered by the effort required to drive them through the thinner strata of hard gravel spoken of, Nova Scotia spruce pine was adopted. This timber is very tough and difficult to split. "Bands" and "shoes" were discarded, being found unnecessary, and, as affording a better base, the toe of the pile was not pointed. The piles were from 12 inches to 18 inches in diameter at the butt, and from 46 feet to 24 feet long, when "cut off." The longest piles were of course used nearest the river, and were the first driven. The shortest piles were under Pier No. 2, and the last driven. The pile driving was finished in September, 1870.

A steam "pile-driver" was used, with a 2,000-lb. hammer, and a 32-feet "drop." The only test as to the stability of the pile when driven was to persistently hammer away, until, after the repeated trials, it was found impossible to force the pile any further.

There were eighty-four piles under each arch pier—four rows of twenty-one piles each—spaced two feet apart from centres. These piles were cut off two feet below low-water mark, and the material excavated from between them to a depth of two and one-half feet below the heads, and this space filled with concrete. A sixteen-inch hemlock grillage, made up of two tiers of eight-inch stuff, from twelve to sixteen inches wide, was bolted to the tops of the piles, and all interstitial spaces likewise filled with concrete. Immediately supporting the foundation masonry was a platform, built of two courses of 3-inch hemlock plank,

spiked diagonally to the grillage and crossed. The dimensions of the platform were 60 feet by 10 feet, and, including the concrete, it was 4 feet 4 inches thick. The foundation masonry under the pier was limestone rubble work, very large and well-shaped stones being used.

The greatest load that can be assumed as being carried by the piles under any one of the arch piers is 2,000 tons; and this load, distributed over eighty-four piles, would give about twenty-four tons per pile, little more than *one-half* their safe load, assuming the piles to be driven to a solid foundation.

And just here we might mention that under the mischief-making Pier No. 2 the piles were driven the same as they were under the other piers—until they would penetrate the soil no further. The platform on the piles was a very stiff one, and a light load was imposed upon each pile. When the foundation was completed there was no reason to doubt its stability, for it was supposed that the hard driving to which the piles were subjected would more than counterbalance the absence of "test-well" exploration, prevented by want of funds. The mechanical force of the blow of the hammer was more than the permanent load upon the pile. The material through which the pile was driven was, from the very surface, impervious to water, and the lower portions of a nature to guarantee a very considerable frictional value. Yet, notwithstanding all this seeming safety, the piles under one end of Pier No. 2 did, finally, sink, after supporting their full load for six or seven years, for reasons given further on in this article.

As previously mentioned, the writer left the bridge upon the completion of the work in 1876, and has since been employed on other and distant work. Having his attention called to the sinking of one of the arch piers by articles in the public press, he visited the bridge on February 7th, 1878, and found that the trouble was all confined to the north end of Pier No. 2, all other portions of the arched approach being in perfect condition. Pier No. 2, it is reported, first began to show signs of failing in the early part of 1877. In about one year—to the date first given—the north end of the pier had settled twenty inches below its normal position. The south end practically stood firm, the south coping and handrailing showing no disturbance whatever in line or level. Owing to the stiffness of the platform, the arch pier and its foundation masonry was sinking as a mass, none of the mortar joints in the pier being cracked, either horizontally or vertically. The skewback

preserved a straight line, as shown by the heavy black line on Plate XXXIII. The foot of the pier had pushed about six inches to the south, thus making it appear as if the platform and foundation and pier masonry were revolving about a point located in the south face of the approach, above the springing line of the arch.

A single crack was apparent in the intrados of each of the brick arches adjoining the sinking pier, making an angle of about sixty degrees with the plane of the springing line. These two cracks were confined to the northern half of the arches, and were not more than half an inch wide at their widest point. The southern half of the arches appeared to be perfectly sound.

Over the north end of Pier No. 2 the mortar joints in the spandrel were badly crushed, and some of the face stones broken, under the pressure. The two coping courses, and with them the footwalk curbing, and one-half the roadway, had settled twenty inches at the lowest point, the settlement taking the form of a regular curve, and extending each way from the centre line of the pier to a point nearly over the crown of the adjacent arches.

On Saturday morning, February 9th, 1878, workmen came upon the ground to commence the much needed repairs. One party began to frame struts, to be set up under the sinking arches, on some plan unknown to the writer. But another gang, before these struts were even partially in place, with very questionable wisdom, commenced to remove the flagging, curb and Belgian blocks over the northern half of Pier No. 2.

According to the statement of the men engaged upon this work, they found this material subjected to great pressure, and, naturally, they experienced great difficulty in displacing it. They say this was done with the idea of reducing the rate of settlement and gaining time, by getting rid of some of the superincumbent weight.

As to what "this rate of settlement" was, having no personal knowledge, the writer can only quote from a published letter of the Survey Department, viz.: "Levels taken six days prior to February 10th did not "show any appreciable difference from those taken three days before; "but from the fifth to the eighth the settlement per *day* increased from "three hundredths of a foot to seven hundredths of a foot. On the "afternoon before the fall it sank from two to three hundredths of a "foot per *hour*, the last twenty-four hours amounting to six-tenths of a

"foot." The words "day" and "hour" are not italicized in the original letter, but show a very sudden and rapid increase in the settlement on Saturday afternoon.

About 7 o'clock on Sunday morning, February 10th, the crippled arches gave way at the haunches and fell; and, there being no centres, ties or supports of any kind under any of the arches, the seven sound arches immediately followed, all falling toward Pier No. 2.

The masonry in the approach was very good, as the ruins testified; portions of the brick arches 10 and 12 feet wide, and 30 feet long, fell to the ground, a distance of about 30 feet, without shattering; some smaller pieces of the arch were turned completely upside down, showing the intrados on top as they lay.

A short time after the fall "test-wells" were, after much trouble, sunk to the rock around Pier No. 2, and from those employed on this work the following data were obtained:

Thirty feet north of Pier No. 2 the rock was found, thirty-three feet below the surface of the marsh; at the north end it was forty feet to the rock; while at the south end of the pier the deposit was thirty-six feet deep to the bed rock; thus making a depression in the profile of the rock just under the north end of the pier. Covering this depression was a bed, about three feet thick, of soft mud, mixed with gravel, and over this a stratum, seven feet thick, of very hard gravel and tough clay. At the south end of the pier there was no mud, and the gravel stratum was reduced to a thickness of eighteen inches. From the gravel to the surface the material was mud—quite soft at the surface, but passing into a tough clay some time before it reached the gravel. This upper mud stratum was impervious to water, no pumping having been required in laying the original foundation, unless a gravel vein was tapped or water came in from the surface. An inspection of the accompanying plan will make the above more clear.

These soundings make plain the original cause of the failure, and explain one of thé "mysterious" features of the case—why the pier should sink, after standing for six or seven years.

Unfortunately, the exact length of each pile, as driven and cut off, cannot now be ascertained, for the contractor was paid "per pile," and not per "lineal foot." But it is a matter of record that the piles under Pier No. 2 were from 28 feet to 30 feet long as hoisted into the "driver," and that the "average length" of the 84 piles, as cut off, was 25 feet.

The want of this exact data makes it impossible to show in the diagram how near the mud the piles at the north end were actually driven.

The writer's theory to account for the stability of the structure for a period of years before failing, is as follows: That at the south end of Pier No. 2 the piles were driven almost, if not quite, to the rock, there not being a sufficient thickness of hard material to prevent this being the case, and, as a consequence, that end stood firm; but at the north end of the pier, under the heavy driving, the piles penetrated almost *through* the hard stratum, say to within two or three feet of the upper limit of the "mud pocket." When this point was reached, the direct and frictional resistance was sufficiently great to prevent further penetration. Just what this frictional resistance was, can, under the circumstances, be only estimated. Rankine's rule, would give, as here applied, about 8 tons per pile. Be this what it may, from the nature of the material, it must have been considerable; and events prove that there was a sufficient amount of hard material between the mud and the toe of the pile to support the structure for a long time.

But the tremor in the piles produced by the heavy and constant travel over the approach was in this case an element of destruction. This tremor had a tendency to loosen the pile from the impervious material into which it was driven, and allow the surface water to slowly find its way down along the pile. In time this water would "lubricate" the pile and destroy the frictional value of the mud, and constantly add *additional* load to that originally carried by the toe of the pile; which original load, we will say, was in this case 16 tons. Possibly, before the whole of the 8 tons (previously carried by friction) was added to the 16 tons already on the toe, the safe bearing value of the hard crust was reached, and the structure began to settle into the softer material beneath it, shown in "black" on the plan. Whether the piles punched through the crust, or whether piles and crust sank together, cannot be ascertained.

As the "mud pocket" was there, it was unfortunate that it did not extend equally under the whole pier; for in that case the piles might have gone down vertically and preserved the horizontal line of the pier. Judging from the condition of the brick arches just before the fall, and from the fact that the rubble haunching was carried up almost to the line of the crown of the arches, the pier might, under this condition of affairs, have sunk much lower than it really did before destroying the

stability of the arches. As it was, one end of the pier sank and the other practically stood firm. Owing to the stiffness of the platform under it, the arch pier and foundation masonry did not crack vertically, as would have been the case with a weak platform under these conditions. But the platform and pier on it remained intact, one end of the mass merely settling lower than the other. As the piles were fastened to this platform, the effect was to throw out of the perpendicular all those piles that were on hard bottom, pushing their heads toward the south. As the upper strata of the soil at that point were soft and afforded little lateral resistance, it was only a question of time when the angle made between the platform and a horizontal line should become sufficiently great to overthrow the piles.

An examination of Pier No. 2, after the fall, showed that it had settled into the soft surface material almost vertically—the springing line being about 12 feet at the north and 6 feet at the south, below its normal position; thus proving that it had overthrown the piles and settled down upon them. The pier was still unbroken, but had pushed several feet to the southward. The spandrel walls had fallen vertically, and the north spandrel, with the skewback and a considerable portion of the ringstone and brick arches still attached to it, extended out some distance over the pier.

In closing this article, the following comments may not be out of place:

The wisdom of adopting segmental stone arches over such an unstable soil, instead of iron plate girders, has been questioned. But this was a question of foundation, and not of superstructure. Stone arches have been standing on pile foundations for centuries without failing, and it was supposed that it could be done again; that the destruction would have been less in this case, had iron girders been used, no one will deny.

But stone arches having been adopted over treacherous ground, nothing but the most searching examination into the nature of the underlying material, at any cost, should have satisfied the engineers. Economy in sinking "test wells" too far apart might in this case have been as fatal as depending entirely upon "sounding with an iron rod."

Pier No. 2 should not have been allowed to settle for nearly a year, and to the extent of twenty inches, before some examination was made into the material beneath.

Precautions should have been taken to secure against accident that portion of the work still uninjured. Twenty-four hours hard work

would have done this. This neglect alone cost eighty per cent. nearly, or about $120,000 of the total loss.

In the opinion of the writer, it was very unwise, to say the least, to *commence* repairs by disturbing the material overlying the sinking end of Pier No. 2. Under the form this material had assumed, it was acting as an inverted arch, and its tendency was to relieve the failing pier of some portion of the superincumbent weight. The sudden and rapid increase in the rate of settlement after this material was disturbed, as shown in the letter quoted, seems to bear out this theory. Repairs were commenced just twenty-four hours before the fall of the arches, and at the point mentioned; and in the latter half of that twenty-four hours ("the afternoon" of Saturday, in fact,) the pier sank at least as much as it had in the preceding ten or twelve days.

While the writer assumes all the responsibility belonging to him, as engineer in charge of the original structure, he prefers having it distinctly understood that he has had nothing whatever to do with the bridge since its completion in February, 1876. Since that time the work has been in other hands.

General Description of the South Street Bridge.

This bridge crosses the River Schuylkill, in the City of Philadelphia. The centre line of bridge commences at the intersection of Chippewa and South street, on the east bank of the river, and making an angle of 35 degrees at the east bank, crosses the river—marshy land beyond and two railroad lines—to the intersection of Thirty-second and Pine streets, on the west bank, a total distance of 1 976 feet, with a width of 55 feet at all points except the draw span, which is 36 feet wide. The eastern approach is made up of 380 feet of heavy sandstone ashlar retaining walls, three "flue" arches at the angle, and a granite abutment at the river 45 feet long by 60 feet wide. The river is crossed by two iron-fixed through spans, each 195 feet 8 inches long, and 36 feet between trusses, and a draw span 198 feet 2 inches long and 23 feet between trusses. The river piers are iron cylinders filled with masonry, and sunk to the bed rock by the "Plenum pneumatic process." The piers for the fixed spans are two cylinders 8 feet in diameter, 36 feet apart from centres, with cast-iron ice-breakers, filled with concrete. The eastern pier is bolted to the rock, 32 feet below high tide; the western pier, 46 feet below high tide.

The draw is carried by nine cylinders, a central one, 6 feet in diameter, and eight others arranged in a circle, each 4 feet in diameter, all tied together by iron beams and sway-braces, and protected by a surrounding cribwork of wood filled with stone. The western arched approach is fully described in the preceding account of the wreck. The railroads are crossed by three iron spans, of a total length of 245 feet. A granite abutment and 75 feet of granite retain wall completes the structure.

The only pile foundations used are under seven of the arch piers and the main abutment west of the river. All other foundations are on platforms of timber resting on gravel.

Note.—The South Street Bridge was authorized by act of Pennsylvania Legislature, finally approved May 3, 1869. By this act a Commission of twenty-two men was created to supervise and control the work, the Chief Engineer and Surveyor of the City of Philadelphia to direct the building of the bridge. On March 1, 1870, the contract for erecting the bridge was awarded to John W. Murphy, C. E.—Moses A. Dropsie, Esq., being the President of the Commission, and Strickland Kneass, C. E., being Chief Engineer and Surveyor of the City. Competitive plans had been previously advertised for, and the plan submitted by J. W. Murphy was adopted by the Commission. The contract price was a round sum of $777,800 for the completion of the bridge. A supplemental contract with Mr. Murphy for the "Draw Fender" raised this sum to $842,800. Work was commenced March 16, 1870. August 12, 1870, D. McN. Stauffer was appointed Engineer in charge for the Commission. In March, 1872, Mr. Kneass resigned his office, and Samuel L. Smedley, the present incumbent, was elected by councils, March 14, 1872, to succeed him as Chief Engineer and Surveyor of Philadelphia. In September, 1874, Mr. Murphy, the contractor, died, and the work was completed by his executors.

On November 30, 1875, the bridge was opened to pedestrians, and formally opened to travel on February 17, 1876.

In accordance with the peculiar municipal organization of the City of Philadelphia, the Department of Surveys merely designs and superintends the erection of all bridges. After the bridge is finished the "Department of Highways" takes sole charge of it and is responsible for all damage caused by neglect to report trouble to the Survey Department, and should ask for plans and estimates for such repairs from the Survey Department. It thus seems that the Department of Highways was responsible, at this bridge, for some portion of the delay in making the proper repairs.

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(Vol. VII.—September, 1878.)

DISCUSSIONS.

(*a*.) On Cements, by DON J. WHITTEMORE and F. COLLINGWOOD.

(*b*.) On Nomenclature of Building Stones and Stone Masonry, by J. FOSTER FLAGG, J. J. R. CROES, J. P. DAVIS, F. COLLINGWOOD, J. VEAZIE and E. P. NORTH.

DISCUSSION OF PAPER CLII.—Notes and Experiments on the Use and and Testing of Portland Cement.*

DON J. WHITTEMORE.—To the engineer about to institute a series of experimental tests of cement Mr. Maclay's paper is of preëminent value in directing an observance of conditions often neglected heretofore.

That low temperature arrests in a marked degree the indurating process, as it does chemical action generally, is well known to many practical engineers. When greater care is taken, not only in regard to the kind of water used, relative fineness of cement, and degree of temperature, of which the paper under consideration specially treats, but as regards a uniform method of manipulation and testing, the subject will then, and not till then, be shorn of the many contradictory results reached by different experimenters, and data will accumulate from which

* Vol. VI. page 312, (December, 1877).

to form correct theory, or, rather, remove the subject from the domain of theory into that of fact.

In my own experimental inquiry in relation to cements, extending through several years, I find that nothing satisfactory can be done without first taking every precaution and refinement in measuring, weighing, and manipulating that can be devised in preparing specimens, and that the resulting tests are of little relative value, unless all specimens are exposed to the same temperature. My own observation leads me to the belief that the action of heat and cold upon mortars of American hydraulic cements is more erratic in character than upon Portland.

The few tests I have made with Portland cement corroborate the deductions of Mr. Maclay as to the weight and fineness of the cement in relation to the tensile strength. I have one example, in which I employed the same sieve he uses (2, 500 meshes to the square inch), which I will refer to. But first I desire to say that I have long since discarded the method of weighing the struck bushel, no matter how filled. By weighing one cubic inch of cement, measured in a mould of that size, under a pressure of 32 pounds, and taking the average of six or eight such weighings, I can arrive at a comparative weight that will be correct to within $\frac{5}{10}$ of one per cent. The amount not passing the sieve, in the example I give, was 13 per cent., and the moulded samples were kept in air at a temperature of about 70 degrees, Fahrenheit, for 24 hours, then placed in water of 58 degrees temperature for six days, and tested immediately after removal, with the following results :

American Portland.	Weight of one cubic inch.	Tensile strength per square inch.
Unsifted..............	$440\frac{8}{10}$ grains.	323 lbs.
Sifted	$426\frac{6}{10}$ "	298 "
$\frac{1}{2}$ screenings and $\frac{1}{2}$ siftings..........		331 "

The portion rejected by the sieve when moulded clear appeared to be comparatively inert, but on *regrinding same,* so that it all would pass the sieve, it gave a tensile strength of 352 pounds at seven days. I take it that the regrinding simply allows the active particles of matter to come in closer contact, and within range, I may say, of their molecular affinities. In manufacturing Portland cement it is not possible to subject all the matter to a uniform degree of heat. The result is that a portion of the compound is underburned, and in grinding pulverizes

easily, while that portion approaching vitrefaction is harder and more difficult to grind, and from it a large portion of the coarser particles is derived.

Too great a degree of heat in burning Portland appears to free some of the lime from silica and other matter of the compound, in which condition the free lime comports as ordinary lime, causing cracks, swellings, &c.; hence through the desire to manufacture an article that will develop great tensile strength at seven days, by high torrification, the result is often the production of a cement possessing these objectionable qualities.

Opinions differ as to the effect of salt, sewer, and fresh water upon cements. I am inclined to the belief that water which has absorbed the most carbonic acid, without matter in solution deleterious to cement, will always be found the best for immersion, in hastening the indurating process, and I can corroborate this statement by many examples within my own experience. Take the example of sewer water immersion, given in the paper under consideration. This water must have been well saturated with carbonic acid, and the best results were attained by the use of it; at the same time it can readily be seen why dirty or sewer water may be injurious for making the initial mixture; for there is engrafted into the composition, whether there exists affinity or not, all the injurious elements of the contaminated water, while in water of immersion molecular affinity makes the selection demanded.

Water, or vapor of water, appears to be the vehicle that transports carbonic acid to the molecules in cement, and even in Portland, as I have proved in repeated instances, the indurating process is hastened largely, after the first act of hydration, by the absorption of carbonic acid. This is eminently true of many varieties of American hydraulic cements.

Though not strictly applicable to the subject under discussion I will give an example of the effect of carbonization upon one variety of American hydraulic cement.

I placed several briquettes of pure cement in a bath of water vapor constantly saturated with carbonic acid, for a period of 52 days. Similar specimens were made and immersed in fresh water for the same period of time, and then all were broken with average results as follows:

Tensile strength, fresh water samples........ 156 lbs. per square inch.
" " carbonic acid bath samples.. 345 " " " "

The tensile strength of cement has become, to quite an extent, the favorite measure of quality. In ordinary use it is subjected to the direct strains of crushing, breaking, and adhesion, and it is my opinion that no system of testing is complete which does not give these values. From my experience, I judge it to be as impossible to determine the resistance of cements to strains of these kinds, from the known tensile strength, as it is to determine the resistance of cast-iron to crushing, from the known tensile strength of wrought-iron. Cements differ as much in ratio of capacity to resist the several strains mentioned as do the several qualities of iron.

For the purpose of securing uniformity in making cement tests, I believe it would be eminently proper for this Society to recommend a code of rules to be observed by experimenters. Specimens can be made much smaller than those usually moulded, and by so doing insure more perfect form and manipulation, with less labor; which will give, in my opinion, more uniform and trustworthy results. Many of my most satisfactory and reliable tests have not required much more than a thimbleful of cement to each specimen. Great pains, however, were taken to have what was used a fair exponent of quality.

Every careful fabricator of mortar has his own personal equation of manipulation. To secure results of a reliable character from which to determine an empirical formula that will approximately express the activity and energy of cement mortars during the period of induration requires the manipulator to have a delicacy and rapidity of touch and the faculty of rapid and careful weighing and measuring not possessed by the ordinary laborer, who is often delegated to perform this part of the work, and to this neglect I attribute a large share of the anomalous results of many experimenters.

In the endeavor to gather data from which to frame such empirical formulæ, I have found it necessary to test samples immediately on their removal from the water, otherwise anomalous results would often occur, such as are shown in Mr. Maclay's Table 14.

Using every precaution in manipulation, exposure and testing, to secure uniformity, the data so gathered warrant me in saying that for at least two years cement mortars never retrograde in tensile strength, but increase approximately in proportion to some root of the age. For a fair quality of Portland, the index of this root is ten (10), and the expression becomes

$$S = A \sqrt[10]{T} \text{ in which formula}$$

S equals the strength at age T, and
A is a factor determined experimentally by dividing the known strength at any age of sample by the tenth root of its age.

I find that many good and bad cements are quite irregular in their rate of induration up to about the 21st day, beyond which time they increase in a ratio that can be approximately formulated; therefore to give a fair judgment of quality, tests at seven days are not always reliable. The value of A in the above formula should be derived from tests made on or after the twenty-first day after fabrication of test samples.

For the purpose of illustrating how close this empirical formula accords with observed tests for the first two years, I extract from the work of John Grant on the strength of cements, page 122, table 18, the following, converting his time of exposure of samples to days, and his strength measure to pounds per square inch.

The last column is deduced from my formula:

$$S = 303. \sqrt[10]{T}.$$

Age in days.	J. Grant's observed strength.	By above formula.
7	363	368
30	416	426
91	469	476
182	523	510
274	542	531
365	547	547
·730	589	586

For further evidence of the correctness of this formula I refer to Grant's table 2, Series A, for hand-made samples, to table 19 and to table 21, samples in fresh water, to which the formulas

$342. \sqrt[10]{T}$,

$267. \sqrt[10]{T}$,

$356. \sqrt[10]{T}$, respectively, approximately apply.

Several tests of my own indicate in a marked degree the correctness of the formula given, when applied to Portland cement of a good or fair quality.

I have a record of tests on Phinney's (American) Portland Cement, purporting to have been made by B. Williams, C. E., of Chicago,

who has the reputation of being a careful experimenter, which are as follows :

Age in days.	Observed tensile strength.	By formula $S = 240.\ \sqrt[10]{T}$.
7	280	291
30	333	337
91	369	377
182	415	404
274	416	421

showing that the formula agrees in this case with observation, within 3 per cent.

The formula does not apply even approximately to the results shown in Mr. Maclay's table, except, perhaps, in the instance of Saylor's cement, the formula for which appears to be $300\ \sqrt[8]{T}$.

I quite agree with Mr. Maclay that the anomalous results shown in this table are to some extent caused by allowing the samples to dry in the open air the last day before testing.

I find that some of the poorer qualities of Portland acquire tensile strength proportional to the 7th or 8th root of the age, with a low factor. The very best I have examined increase as the 12th root, with a high factor.

I desire to express the opinion that if the strength value of Portland cement is to be determined only by tensional strength, the sooner the usual seven-day tests are discarded the better, and substitute tests at 20, 40, and 60 days, and then demand that a fair quality shall not indicate a factor, and an index of root 10 per cent. less than is given in the formula.

$$S = 280.\ \sqrt[10]{T},$$

S representing strength in pounds per square inch, and T age in days.

The expression $S = A\ \sqrt[x]{T}$ I find to be of very general application, both for representing tensile and compressive strength, not only of Portland but of Roman cements, when once the co-efficient A and index X are experimentally determined.

It follows, therefore, that the probable activity of a cement can be determined, for a unit of time at any period, by the formula

$$\text{Activity} = \frac{S}{T \times X}$$

From tests and appearances of samples from time to time, I think it possible that induration is the result of, at least, three operations; first,

an initial act of hydration, like hardening in water of plaster of Paris, and of some calcined rocks not properly classed among the cement yielding stones, which substances, under the conditions named, perform the act of setting, and acquire an ultimate crushing strength of, perhaps, 200 or 300 lbs. per square inch only; second, formation of silicate and aluminate of lime, as in the case of Portland cement, and silicate and aluminate of lime, and of magnesia, as in the case of Roman cement; third, carbonization, requiring great length of time, probably centuries, for complete results.

Analysis of ancient mortars show that but few have absorbed carbonic acid to the point of complete saturation, and these few are, apparently, as hard as flint.

With some abnormal cements, after the first act of hydration, their further hardening seems to be arrested, in a marked degree, from after a few hours to, occasionally, twelve or fifteen days, after which violent activity commences, a large afflux of "laitance" takes place, and from this time on, up to about two years, the induration proceeds with a regularity and ratio approximately indicated by the formula given.

F. Collingwood.—I wish in this discussion to give the results of some recent experiments on American cements and bricks. These experiments were made at the works of the East River Bridge by Mr. Abbott, one of the assistants, with the intention of conforming, as near as might be, to the ordinary surroundings of temperature, &c.

The only exception to this rule was that he tried to limit the quantity of water to that amount which would produce in each case the best result. This was found to vary with every lot of cement, even from the same maker. That which in one case would make a clean hard briquette, would in another not give any coherence when rammed. The variation seemed to depend as much upon age as anything.

The percentage by weight of water used is given on the diagram, Plate XXXIV. This was just enough to make the mass slightly moist, after which it was rammed in the moulds. Mr. Abbott estimates that about one-half more water would in each case give a mortar of the right consistency for use.

The fineness of the cements is also noted on the diagram, and indicates the percentage which passed through a sieve of 2 500 meshes per square inch.

The temperature of the air was from 40° to 70°, and of the water used 50° to 60° Fah.

The briquettes were made 2″ × 1½″ in the breaking section, with ends enlarged to fit the clamps in the testing machine. In *compression* a portion of the *same* specimen was crushed, so that the diagram gives the means of a direct comparison between these strains. The size was 2″ × 2″ × 1½″ thick.

Each set of curves represents the results of 40 individual tests, 10 being broken at 24 hours, 10 at 7 days, 10 at 14 days, and 10 at 21 days setting, the briquettes in each set being made at the same time, and from the same barrel.

The lower line gives the minimum strength of the set, the middle line the average, and the upper line the maximum.

The results for the "F. O. Norton," and "N. Y. and Rosendale" brands show a fair average of the cements furnished us for use. But *one* barrel of each of the others was experimented upon, so that it would be unfair to draw comparisons.

The chief point of interest in the curves is the rapid increase in strength up to about 7 days time, then the depression in almost all cases up to the 14th day, and then the marked and steady increase. It would be interesting to know whether this is accidental or real. The first period would seem to indicate the close of the formation of hydrates, and the next rise the beginning of the formation of crystals of silicate of lime, &c.

The curves show clearly also that 24-hour tests are no certain criterion as to the ultimate strength of cements. So far as these experiments go, they show also that there is no certain ratio between tensile and compressive strength, or in other words that the ordinary tests are of little value as to determining the true relative merits of different cements.

By my request, some tests were also made of the strength of brick both in compression and tension. The table is given herewith.

The bricks were selected to give a fair average of "good Haverstraw stock brick," not the hardest burned.

No packing was inserted in the machine between the bricks and the compressing surfaces; so that the strength in compression represents the case of imperfect beds, &c., although it was found that it made but little difference.

The test of whole bricks on *end* gave a minimum strength per square inch in 10 tests, of 1 600 lbs., a maximum of 3 060 lbs., and an average of 2 065 lbs.

Ten half-bricks were then broken by compression on the *side*, giving minimum strength per square inch of 2 900 lbs., maximum of 6 400, and average of 4 612 lbs.

Ten half-bricks were then broken on the flat, and to our surprise the figures were minimum, 2 669 lbs.; maximum, 4 153 lbs.; average, 3 371 lbs. The less amount is probably due to imperfect bearing surfaces.

Comparing these figures with those given for cements, we see they are fully equal to the best at the end of 21 days, and they represent a fair average of the bricks received by cargo.

As to *tension*, 12 bricks were carefully cut to fit the cement testing machine, and the bearing packed so as to avoid cross strains. The results were, minimum strength per square inch, 90 lbs.; maximum, 358 lbs.; average, 168½ lbs.

This again is about equal to the best cements (omitting the Portlands), after 21 days setting.

The lesson to be drawn is the old one that brick-work has considerable strength as a beam, and such strength can safely be relied upon where the work is well done.

In cutting off some of our walls, laid in cement mortar, which had been laid up from four to eight weeks, the adhesion was such that bricks would break in two in every case. This would not be true with partly filled joints, but is invariably true with good work.

Experiments on the Strength of Bricks.

Compression.

Position.	Load per □''.	Position.	Load per □''.
Whole brick on end ...	1 600	Half bricks on flat side..	3 461
	2 031		3 146
	1 956		3 800
	2 285		3 331
	2 412		4 153
	1 524		2 707
	1 832		2 669
	2 019		3 923
	1 931		3 423
	3 060		3 101
10	20 650	10	33 714
Average.	2 065	Average.	3 371.4
		Tension.	
		Brick pulled apart.....	122
			100
½ bricks on small side...	5 080		358
	6 400		124
	5 400		144
	5 620		106
	2 900		204
	4 320		124
	4 540		337
	3 780		204
	4 360		90
	3 720		108
10	46 120	12	2 021
Average.	4 612	Average.	168½

Discussion of Paper No. CLI.*—Nomenclature of Building Stones and of Stone Masonry.

J. Foster Flagg.—I consented to take part in the discussion of the valuable paper upon the "Nomenclature of Building Stones and Stone Masonry," without duly considering, perhaps, how little of merit I could contribute to it,—especially with the little time latterly at my disposal.

The subject is quite an important one to us all, as there is scarcely any branch of our profession that does not involve in its practice the construction of masonry. Like many others, I have frequently been at a loss to know exactly the character and class of masonry in the descriptions and specifications of different works from the variety of meaning attached in different localities to the same technical term, or the different terms used to describe very similar kinds of work ; and it is very desirable, to say the least, that a uniformity of practice should be determined upon, so that the same term will always convey the same meaning to all, whether in discussions and papers, or advertisements and specifications.

This naturally does not diminish the desirability in all works of importance, of the use of sample blocks, in order that contractors may clearly understand the degree of finish demanded in stone cutting, and that disagreements and contests upon this point during the progress of the work may be avoided.

I notice that the master car builders of our railroad corporations have experienced the same trouble in the confusion of names applied in different sections of the country to the several parts of a car or coach, and that they have taken a step in the right direction, in the compilation of a dictionary of technical terms used in car building, showing all the different significations attached to the same term, and the various terms used for the same part. If they can follow up this work by the simplification of their nomenclature, it will contribute much to the clearness of discussions in their conventions, and to the general understanding of their papers and reports.

A similar dictionary of terms used in all branches of American engineering practice would be of exceeding value, if some one could be found with sufficient courage to undertake its compilation, and to do it thoroughly.

* Vol. VI, page 297 (November, 1877).

In this paper the description of tools used in stone cutting impressed me as being very clear and complete. The section upon stone cutting is also very good, and, perhaps, sufficiently classified for engineering purposes ; but, under the head of masonry, it seems to me that something more is needed—that the classification is too general.

As an example, certain masonry on Western Railroads is sometimes classed—very indefinitely—as *box* masonry, from its being used in box culverts. This very frequently means simply rubble work, but it often, with the soft sandstone used in certain sections, becomes almost the "squared stone" masonry of this paper—being sometimes even laid in regular courses—and yet it can hardly be fitly classed thus ; the stone is very soft, easily split, and quickly dressed roughly with the face hammer; so the blocks merely roughly knocked into shape with the hammer, are far from being rubble, and yet are much inferior to the "square stone" described (p. 300), under the head of stone cutting, they having no joints that can properly be spoken of as dressed. The character of the work may be better appreciated by the statement that it cost, a few years since, at least 30 per cent. less than abutment masonry with rubble backing, the facing of which in the East would be considered only ordinarily good quarry faced squared stone masonry, with no pitch facing or drafting, except on the angles of the abutments. I give this only as one example.

It is much easier to criticize than to originate, and I confess that I have no substitute to offer at present for this classification—it needing much more study than I have been able to give the subject. I only suggest, therefore, that a little more elaboration in the classification would be desirable.

J. J. R. Croes.--The advisability of making a finer subdivision of classes of masonry was discussed by the Committee. It was found that it would lead to the enumeration of so great a variety of styles of cutting and laying stones, that confusion rather than preciseness would result.

It is believed that the three main classes of masonry suggested, will be found to cover almost every kind of work used in engineering structures, with the aid of some intelligence and a definite idea of what is wanted, on the part of the person who prepares the specifications.

While on this subject, I would like to ask the members present what each understands by the word "build" as applied to cut stones. I have always understood it to mean the *top bed*, but have recently found that

some engineers understand it to mean the vertical end joint. What is the generally accepted meaning of the term ?

A MEMBER.—It is the upper surface of the stone.

J. P. DAVIS.—I believe that in the New England States it is understood to be the top surface.

F. COLLINGWOOD.—I think that by some it is understood to mean the vertical joint. I am in doubt myself which is correct.

J. VEAZIE.—In specifications for cut stone work on fortifications the word *build* means the upper surface of the stone.

E. P. NORTH.—I have always understood the build to mean the amount that the course raises the masonry, or the vertical height of the stone.